

*Citation for published version:*

Drougkas, A, Verstryng, E, Szekér, P, Heirman, G, Bejarano-Urrego, L, Giardina, G & Van Balen, K 2020, 'Numerical Modeling of a Church Nave Wall Subjected to Differential Settlements: Soil-Structure Interaction, Time-Dependence and Sensitivity Analysis', *International Journal of Architectural Heritage*, vol. 14, no. 8, pp. 1221-1238. <https://doi.org/10.1080/15583058.2019.1602682>

*DOI:*

[10.1080/15583058.2019.1602682](https://doi.org/10.1080/15583058.2019.1602682)

*Publication date:*

2020

*Document Version*

Peer reviewed version

[Link to publication](#)

This is an Accepted Manuscript of an article published by Taylor & Francis in International Journal of Architectural Heritage on 14 April 2019, available online:  
<http://www.tandfonline.com/0.1080/15583058.2019.1602682>

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# **Numerical Modeling of a Church Nave Wall Subjected to Differential Settlements: Soil-Structure Interaction, Time-Dependence and Sensitivity Analysis**

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## **Abstract**

Historic masonry structures are particularly sensitive to differential soil settlements. These settlements may be caused by deformable soil, shallow or inadequate foundation, structural additions in the building and changes in the underground water table due to the large-scale land use change in urban areas.

This paper deals with the numerical modeling of a church nave wall subjected to differential settlement caused by a combination of the above factors. The building in question, the church of Saint Jacob in Leuven, has suffered extensive damage caused by centuries-long settlement. A numerical simulation campaign is carried out in order to reproduce and interpret the cracking damage observed in the building.

The numerical analyses are based on material and soil property determination, the monitoring of settlement in the church over an extended period of time and soil-structure interaction. A sensitivity study is carried out, focused on the effect of material parameters on the response in terms of settlement magnitude and crack width and extent. Soil consolidation over time is considered through an analytical approach. The numerical results are compared with the in-situ observed damage and with an analytical damage prediction model.

## **Keywords**

Masonry; soil-structure interaction; historic structures; finite element modeling; settlement-induced damage

## 39 **Highlights**

- 40           • A masonry church nave wall subjected to differential settlements is numerically modeled, considering
- 41           time-dependent material properties combined with changes in geometry and loading
- 42           • The foundation and the soil properties are directly considered
- 43           • A sensitivity analysis highlights the parameters affecting the cracking pattern and extent
- 44           • The phased analysis results in a much more accurate representation of the observed damage compared
- 45           to a single-phase model
- 46           • An analytical model for the calculation of damage due to differential settlements is expanded and
- 47           compared to the finite element analysis results

## 48 **Notation**

49	$E$	Young's modulus
50	$G$	shear modulus
51	$\nu$	Poisson's ratio
52	$\rho$	mass density
53	$\sigma_t$	tensile stress
54	$\varepsilon_{cr}$	crack strain
55	$\varepsilon_u$	ultimate strain
56	$f_c$	compressive strength
57	$f_t$	tensile strength
58	$G_f$	tensile fracture energy

59	$h$	characteristic finite element length
60	$L^j$	footing half-length
61	$B^j$	footing half-width
62	$D^j$	footing embedment depth
63	$A_f^j$	footing area
64	$A_w^j$	footing side-wall contact area
65	$K_n^j$	footing vertical elastic stiffness
66	$k_n^j$	footing modulus of subgrade reaction
67	$k_n$	wall/colonnade modulus of subgrade reaction

## 68 **1. Introduction**

### 69 **1.1 State of the Art**

70 The analysis of large monumental structures subjected to differential ground movement is a challenging  
71 subject of study. The challenge mainly arises from geometric complexity and sheer size, problems only partially  
72 mitigated by a detailed geometric survey, material property determination and the definition of the applied  
73 deformation load profile through concerted monitoring efforts [1]. It is, however, a worthwhile endeavor in service  
74 of estimating the risk of damage or collapse and designing effective intervention strategies for repair and  
75 strengthening.

76 The discretization of monumental church structures in macro-elements with different stiffness is often  
77 considered conceptually and empirically valid. Macro-elements, such as façades, towers, apses and single naves,  
78 are often separated through insufficient tying and the presence of structural cracking. They are further

79 characterized by different stiffness, weight and internal stresses. Namely, the effects of earthquake action and  
80 differential settlement loading affect different parts of the structure in distinct ways. Therefore, the problem of  
81 structural analysis of large churches can be simplified by pursuing it on an individual macro-element basis [2]. It  
82 is both practical and valid to study certain components of monumental church structures, such as single bays,  
83 naves, façades or towers individually.

84 In a nonlinear finite element (FE) analysis framework, the large dimensions of monumental church structures  
85 built in masonry can render detailed modeling computationally prohibitive and model preparation effort excessive.  
86 As an alternative to nonlinear FE modeling, rigid block analysis has been shown to be capable of reproducing  
87 failure modes in masonry structures subjected to large movement of their supports [3,4]. When failure is mostly  
88 concentrated in the joints, either through sliding and/or opening, as is the case in dry joint masonry and, usually,  
89 in masonry with weak lime mortar joints, rigid block analysis becomes an attractive approach. However, sheer  
90 size and geometrical complexity, due to an irregular bond and the existence of multiple masonry leaves, render  
91 such approaches difficult to implement in large structures. Macro-modeling, therefore, which consists in the  
92 homogenous modeling of the masonry composite, becomes a more suitable alternative. Despite the assumption of  
93 homogeneity, nonlinear macro-modeling of large masonry structures can provide insight into the mechanisms  
94 through which damage arises and expands [5,6].

95 Predictive models for damage estimation and categorization in masonry structures subjected to differential  
96 ground movement have been proposed in the literature [7–9]. These models rely on the determination of  
97 parameters related to the material, geometric and foundation properties of the structure and are mostly used to  
98 evaluate the effects of tunneling-induced settlements. The loading parameters for these models can be determined  
99 using terrestrial or space-borne means, comparing the differential settlement to empirical, semi-empirical or  
100 calculated limits for the determination of the level of damage. The analytical and parametric basis of these models  
101 allows their adoption and modification according to the requirements of a variety of loading scenarios and  
102 foundation types. The application spectrum of such methods extends from damage prediction in individual

103 buildings to vulnerability assessment in entire urban aggregates. This basis further allows the parallel application  
104 with, and direct comparison to, numerical modeling of damage induced by differential ground settlement.

105 In this paper both numerical and analytical models are applied for the analysis of a large monumental structure.  
106 The church of Saint Jacob in Leuven has been the subject of wide and inclusive study over the preceding decades  
107 [10–12]. It is characterized by extensive and developing damage due to differential ground movement. This  
108 damage is well documented and extensive data is available on the profile of ground movement over different  
109 periods. Finally, studies have been performed on its material properties, the stress state of its structural elements  
110 and the properties of the foundation soil. It is therefore a prime candidate for investigation through analytical  
111 simulation and numerical analysis for the purpose of interpreting the damages present in the fabric and the  
112 development of ground movement.

## 113 **1.2 Objectives**

114 The focus of the present paper is the numerical reproduction of the damage patterns observed over time in the  
115 church of Saint Jacob in Leuven, the investigation of their underlying cause, namely the differential settlements at  
116 the site, and the study of the effect of material and numerical analysis parameters in the obtained results. This is  
117 accomplished through a sensitivity study involving the material properties of the masonry structure, the application  
118 of different loading patterns in the form of settlement profiles and the variation of the boundary conditions as  
119 affected by the passage of time.

120 The interpretation of the results of the numerical analyses, coupled with the assembly and evaluation of historic  
121 data, on-site observations and structural monitoring aim at providing insight into the occurrence and development  
122 of structural damage in the monument. A quantitative assessment of the development in time of soil consolidation  
123 under the effect of gravity loads is given, thus outlining the behavior of the monument over an extended period.

124 Further interpretation and quantification of the numerical results is provided through their comparison with  
125 simplified analytical damage assessment models. This comparison allows the evaluation of the applicability of

126 analytical models in monumental masonry structures and demonstrates their potential for the interpretation of  
127 current and the prediction of future structural damage.

## 128 **2. The Case Study**

### 129 **2.1 Layout and Brief Historic Outline**

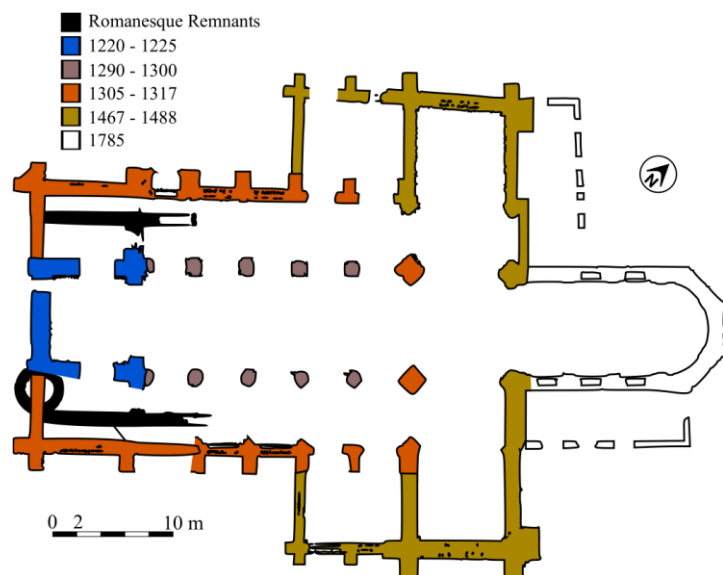
130 Details on the history of the construction of the church may be found in [10], with the main points repeated  
131 here for clarity. Construction of St. Jacob's church began around 1220, with the erection of the tower over the  
132 remnants of an existing Romanesque church. The initial plan called for the church to have a flat timber ceiling,  
133 which was later substituted for a timber barrel vault. The main nave, at its originally intended height, was  
134 completed in the 14<sup>th</sup> century, along with the side naves and their stone vaults, and the bell tower over the crossing  
135 was added in the 15<sup>th</sup>. During the period 1534-1535, an additional level over the main nave was added and masonry  
136 vaults were added in place of the timber vault, which was complemented by the addition of two, possibly four,  
137 pairs of flying buttresses. These alterations resulted in the addition of self-weight not originally anticipated in the  
138 construction of the foundations.

139 First mention of structural problems stemming from differential settlement dates back to at least the 15<sup>th</sup>  
140 century. These problems led to the reconstruction of the side nave vaults. The timber bell tower over the crossing  
141 was dismantled in 1735 due to concerns over its decay. The development of vertical cracking in the pillars led to  
142 the installment of confining steel rings in the early 19<sup>th</sup> century, still present. Further consolidation measures were  
143 taken in the early 20<sup>th</sup> century due to severe cracking in the west wall of the northern transept. In 1963, the entire  
144 church was definitively closed for the public. During the partially executed consolidation works of 1965-1971, the  
145 structure was internally shored using massive reinforced concrete elements and steel profile braces. While only  
146 foreseen as a temporary measure, the shoring members are still present today. Additionally, the side nave vaults  
147 were dismantled for weight reduction. In 2000 the flying buttresses were removed due to their being severely out-  
148 of-plumb, which raised concerns of sudden collapse, and were replaced by temporary steel tie-rods. A  
149 comprehensive structural intervention project, including localized repairs on the masonry a micro-piling



reinforcement of the foundations and reconstruction of the dismantled elements (side nave vaults and flying buttresses) and soil consolidation was launched in September 2018 [13].

A floor plan of the church, along with the designation of the construction phases, can be seen in Figure 1. The construction process, beginning with the tower and following with the arcade, naves, transept, chapels and finally the choir are indicated.



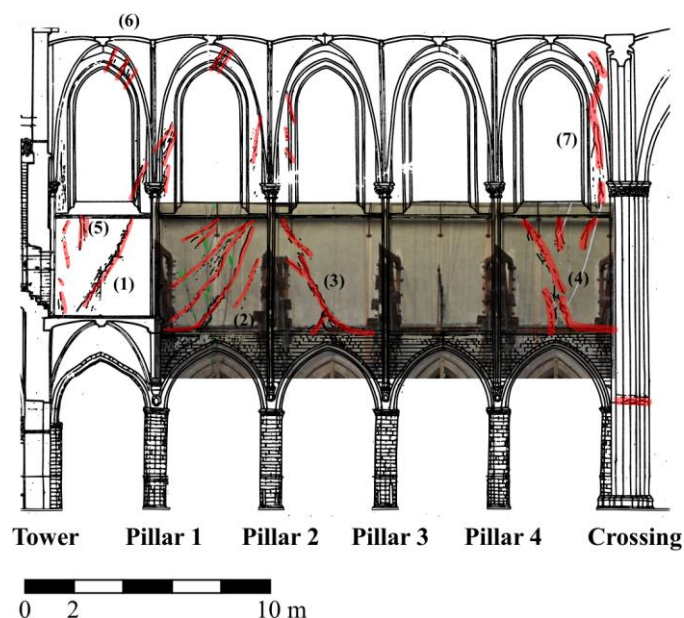
**Figure 1 Church floor plan and construction phases. Adapted from [10].**

## 2.2 Damage Survey

The differential settlements in the church, which are the cause of the clear majority of structural damage, are caused by the building being erected in a swamp area near the river Voer, coupled with the initially unplanned addition of a second and third level, resulting in a severe increase in self-weight. The substitution of the original flat timber ceiling with a wooden barrel vault and, later, a masonry vault resulted in further increase in the self-weight.

The present study focuses on the damage documented in the northern wall of the main nave. An elevation view, the main structural elements and damage, along with the notation used for their designation, is shown in Figure 2. The nave wall measures approximately 26 m in length and 21 m in height. The damage of the nave,

166 consisting primarily of cracks caused by differential settlement, has been documented both with hand drawings  
 167 and photographically during site visits, and more recently using semi-automated point-cloud data processing [14].



169 **Figure 2 Northern wall of main nave: structural element designation and documented major cracks**  
 170 **indicated in red (adapted from a hand-drawn survey of the building's geometry and pathology carried out**  
 171 **by students of the Raymond Lemaire International Centre for Conservation, 1983-1984 [15]). In underlay**  
 172 **a photographic survey of northern nave wall cracks. View of cracks above nave pillars (photo by Pepijn**  
 173 **Szekér, 2018).**

174 The arithmetically designated cracks 2 through 4 of the nave have been photographically documented and are  
 175 shown in underlay in Figure 2. Continuous visibility of cracks 1 through 4 is not possible due to the obstruction  
 176 caused by the organ loft near the western tower. Despite the time passed between the survey in 1983-1984 and  
 177 the photographic survey in 2018 (Figure 2), there does not appear to be any lengthening of the major cracks.  
 178 However, the same cannot be said with certainty about the crack widths, however, since these were not measured  
 179 in the prior case.

180 The widths of the cracks above the nave pillars have been measured by hand. While the external plaster  
 181 presents a crack width of a few mm, the crack width on the masonry behind the plaster is roughly between 10mm

182 and 20mm (i.e. eroded mortar joints), Figure 3. The cracks appear to mostly pass through the mortar joints rather  
183 than splitting the masonry stone units in the investigated area.

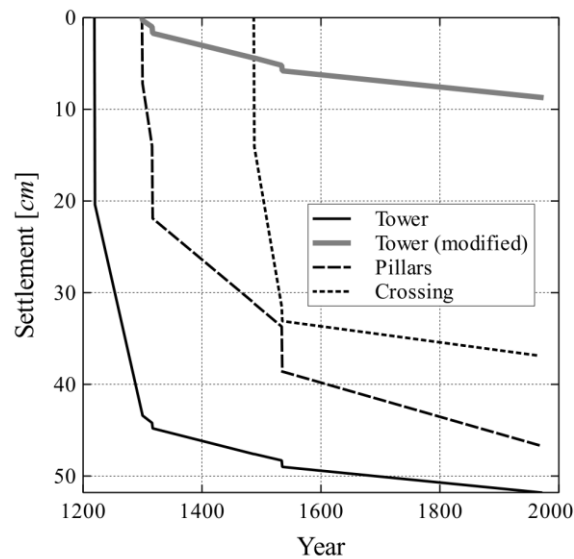


184

185 **Figure 3 Close-up of cracks 4 on main nave wall (photo by Els Verstrynge, 2018).**

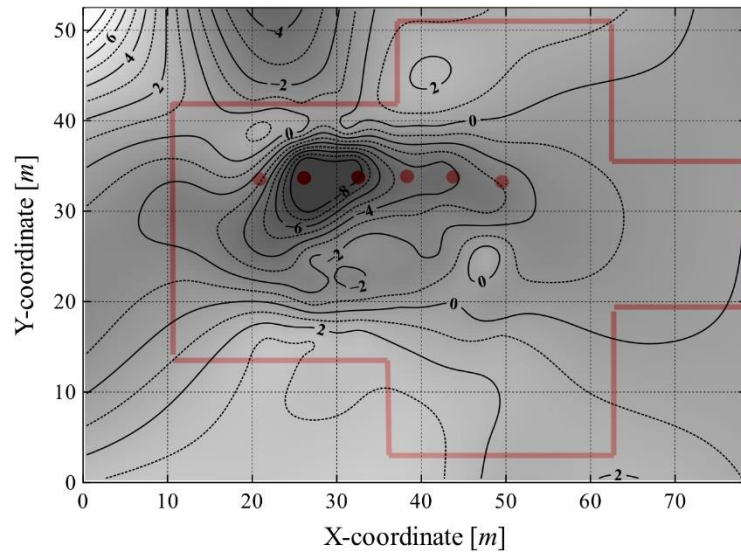
## 186 **2.3 History of Settlements: Estimation and Geomatic Data Processing**

187 An estimation of the soil settlements for different parts of the structure, under different calculated loads, has  
188 been carried out in the extensive studies of the church. These results are presented in internal technical reports,  
189 property of KU Leuven [15]. These settlements are presented for the tower, crossing and pillars in Figure 4 and  
190 have been calculated based on cone penetration tests and evaluation of the soil consolidation progress according  
191 to Terzaghi, Buisman and Koppejan [16]. The results are not differentiated between individual pillars, thus  
192 rendering the calculation of the differential settlement between pillars impossible. Since the tower was completed  
193 before the beginning of the construction of the nave, an additional graph of the development of the tower settlement  
194 is provided. This graph ignores the settlements occurred before the completion of the nave wall (see grey line in  
195 Figure 4).



**Figure 4** Calculated settlements for different structural components of the tower and nave from 1220 to 1970 (adapted from [15]).

Concerning more recent ground movement and the resulting differential settlement of the pillars, levelling surveys conducted over the previous two decades provide info on the development of differential settlements over selected periods of measurements. Contour plots of the settlements over the period 1994-2005 are presented in Figure 5. These geodetic survey results are presented in terms of settlement relative to a point in the choir which is considered, due to the absence of apparent damage, stable. The maximum settlement over this period was measured at the area around pillars 1 and 2 of the northern nave, with areas at the southern nave and northern transept presenting some uplifting. The settlement at pillars 1 and 2 is consistent with cracks 1, 2 and 3 as indicated in Figure 2, although the time of the first appearance of these cracks is not known with certainty. It is interesting to note that all points along the nave exhibited a rather uniform settlement in the period 1994-2000, while in the period 2000-2005 the settlement of pillar 2 appeared to increase at a faster rate than the other points in the nave.

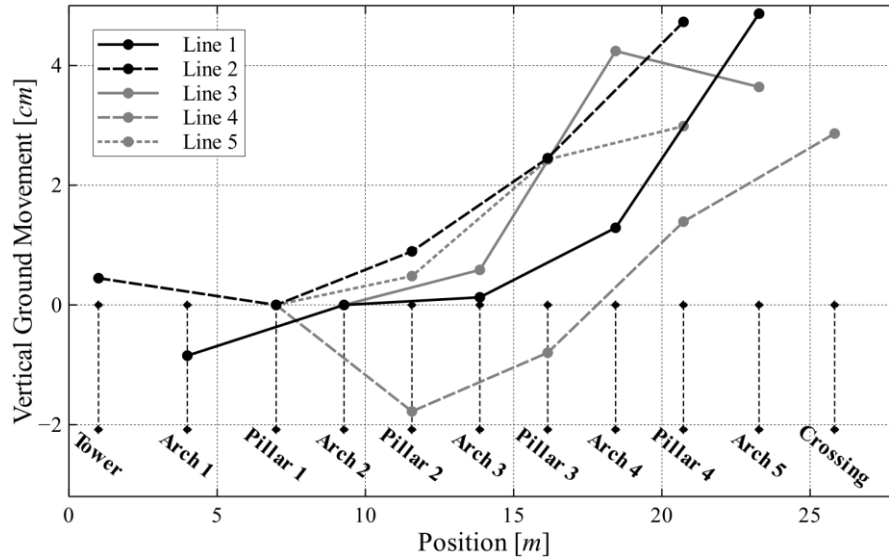


**Figure 5** Contour plot of ground settlement in mm during the period 1994-2005 against church architectural outline. Location of northern nave from tower pilaster to crossing indicated by solid hatch circles.

An additional third approach may be adopted for the evaluation of the differential settlements over the entire history of the church. This is accomplished through the study of the geometric disposition of characteristic architectural features present on the structural elements. This approach is based on detailed laser scanning data acquired in the context of this investigation [14]. The downside of this rough approach is the inability to evaluate the total settlement that each element has undergone: only the final differential settlement can be estimated. Five reference features are chosen for this approach, from top to bottom: 1) the top of the third level arches' voussoirs, 2) the base of the pilaster abacus of the third level, 3) the top of the first level arches' voussoirs, 4) the base of the pillar abacus of the first level and 5) the top of the first level pillar pedestals. Assuming that features 1-2, and similarly 3-4-5, were built at the same period, it follows that they were, in all probability, vertically level at the time of construction.

The measured vertical displacements relative to pillar 1 (lines 2, 4-5) or arch 2 (lines 1, 3) are shown in Figure 6. All architectural features are present in pillar 1 and its neighboring arch span, hence the choice of these elements as a reference, instead of, for example, the crossing column. The obtained profile is similar to the profile found in

226 the recent leveling measurements, with the settlement being mostly concentrated around pillars 1 and 2. This is  
 227 consistent with the formation of cracks 2 and 3 (Figure 2). Differences in the profile at different heights of the  
 228 structure are expected for two reasons: a) differential settlement generally affects the lower parts of the building  
 229 more severely and b) the second and third levels of the nave wall were constructed at a later phase, when part of  
 230 the settlements of the colonnade had already occurred.



231

232 **Figure 6 Differential settlements relative to pillar 1 (position: 6.99 m) measured from point cloud**  
 233 **data architectural feature analysis. Position distance measured from tower pilaster.**

## 234 3. Analysis Procedure

### 235 3.1 Modeling the Nave Wall

236 The geometry of the nave is derived from an idealization of the in-situ geometry in its undeformed state. A  
 237 distinction is made between the three-leaf masonry of the nave wall and the solid stone masonry of the lower part  
 238 of the pillars, each with its own set of material properties. The basic values of the material properties used for the  
 239 numerical analyses, in part determined in previous experimental efforts [10,11] and in part assigned nominal or  
 240 empirical values as proposed in the relevant literature [17,18], are summarized in Table 1.

241 **Table 1 Basic material properties used in numerical analysis.**

	$E$ [N/mm <sup>2</sup> ]	$\nu$ [–]	$\rho$ [kg/m <sup>3</sup> ]	$f_c$ [N/mm <sup>2</sup> ]	$f_t$ [N/mm <sup>2</sup> ]	$G_f$ [N/mm]
Wall masonry	3000 <sup>c</sup>	0.15 <sup>c</sup>	1920 <sup>b</sup>	6.99 <sup>a</sup>	0.10 <sup>c</sup>	0.012 <sup>d</sup>
Pillar masonry	15700 <sup>b</sup>	0.20 <sup>b</sup>	2360 <sup>b</sup>	11.95 <sup>a</sup>	1.00 <sup>d</sup>	0.075 <sup>d</sup>
a: experimentally derived value [10]				c: estimated value [17]		
b: experimentally derived value [11]				d: estimated value [18]		

242 The masonry walls and columns of the nave are modeled using 8-node quadrilateral and 6-node triangular  
243 plane stress elements, an approach suited to the geometric arrangement, element thickness and load orientation. A  
244 macro-modeling approach is adopted for the model, in which the masonry composite is treated as a homogenous  
245 continuum, with no distinction between units, mortar and the unit-mortar interface. The nonlinearity in tension is  
246 modeled using a multi-directional fixed crack model [19]. The model is based on a decomposition of the total  
247 tensile strain into an elastic and a crack component. The crack strain is further decomposed, allowing for the  
248 formation of a number of cracks simultaneously. A Rankine-type tension cut-off is used in pure or biaxial tension,  
249 while the influence of lateral compression is accounted for through a Mohr-Coulomb-type criterion. Nonlinear  
250 tension softening is assumed, governed by fracture energy, according to the expression:

$$\sigma_t = \begin{cases} f_t \left(1 - \frac{\varepsilon_{cr}}{\varepsilon_u}\right)^{0.31} & \text{for } 0 \leq \varepsilon_{cr} \leq \varepsilon_u \\ 0 & \text{for } \varepsilon_u \leq \varepsilon_{cr} < \infty \end{cases} \quad (1)$$

251 where  $\sigma_t$  is the tensile stress,  $\varepsilon_{cr}$  is the crack strain and  $\varepsilon_u$  is the ultimate strain, calculated according to the  
252 expression [20]:

$$\varepsilon_u = 4.226 \frac{G_f}{f_t h} \quad (2)$$

253 where  $h$  is the characteristic length of the finite element. The fracture energy/characteristic length approach  
254 results in mesh objectivity, provided the element length is sufficiently small to avoid a constitutive snap-back. The

255 maximum size criterion, based on the requirement for the initial tangent of the tensioning softening diagram to be  
256 less than the Young's modulus, is satisfied for the chosen element length.

257 While the nave cracks appear to be mostly developed along the joints and not through the units. A detailed  
258 modeling approach based on rigid-block or block-joint models could potentially be employed. However, the  
259 dimensions of the structure are prohibitive for such detail to be practical. The geometric survey would need to  
260 include detailed information on the dimension and arrangement of the outer leaf stones. Further, even if nominal  
261 dimensions and a regular pattern were adopted, the infill would still need to be individually modeled. Macro-  
262 modeling was therefore adopted as a practical solution, nevertheless capable of providing sufficiently detailed  
263 results for the purposes of the paper.

264 The steel rings installed in the 19<sup>th</sup> century were not included due to the lack of data on their material properties  
265 and state of decay. It is not expected, however, that this omission affects the cracking of the nave walls to any  
266 significant extent.

### 267 **3.2 Foundation and Soil-Structure Interaction**

268 Soil-structure interaction is directly considered through the introduction of linear elastic structural interfaces  
269 at the base of the masonry pillars, capable of accounting for normal and shear deformation. The normal stiffness  
270 may be determined according to two distinct approaches: (a) from a calibration effort targeted at reproducing the  
271 settlement profile measured over a given time period and (b) directly from the geometric characteristics of the  
272 pillar footing and the elastic properties of the soil.

273 Both approaches are adopted and compared in the present paper. The values obtained from approach (a) are  
274 presented in the results section. For approach (b), the vertical elastic spring constant for a single rigid arbitrarily  
275 shaped footing  $j$  circumscribed in a rectangle with dimensions  $2L^j \cdot 2B^j$  and embedded in the ground at a depth  
276 of  $D^j$  is equal to [21]:



$$K_n^j = \frac{2GL^j}{1-\nu} (0.73 + 1.54x^{0.75}) \left[ 1 + \frac{1}{21} \frac{D^j}{B^j} (1 + 1.3x) \right] \left[ 1 + 0.2 \left( \frac{A_w^j}{A_f^j} \right)^{2/3} \right] \quad (3)$$

277 where  $G$  is the shear modulus of the soil,  $\nu$  is the Poisson's ratio of the soil,  $A_f^j$  is the area of the footing,  $A_w^j$   
 278 is the total sidewall-soil contact area (equal to the perimeter of the footing times the embedment depth  $D^j$  in case  
 279 of a foundation with constant cross-section) and  $x = A_f^j / (2L^j)^2$ . Division of the spring constant by  $A_f^j$  produces  
 280 the modulus of subgrade reaction for a single footing:

$$k_n^j = \frac{K_n^j}{A_f^j} \quad (4)$$

281 This value is used for the normal stiffness of the interfaces below the pillars, adjusted according to the ratio of  
 282 the base area of the footing over the cross-sectional area of the pillar at ground level. The settlement  $d_n^j$  of the  
 283 foundation for a given normal force  $F_n^j$  is:

$$d_n^j = \frac{F_n^j}{k_n^j} \quad (5)$$

284 The value for the shear stiffness of the interface is calculated by the expression:

$$k_s^j = \frac{k_n^j}{2(1+\nu)} \quad (6)$$

285 according to the Poisson's ratio  $\nu$  of the stone masonry foundation. This value is not determined  
 286 experimentally but is consistent with the material properties of the stone masonry and is numerically more stable  
 287 and less arbitrary than the use of a dummy value that precludes shear slipping at the foundation. This slipping

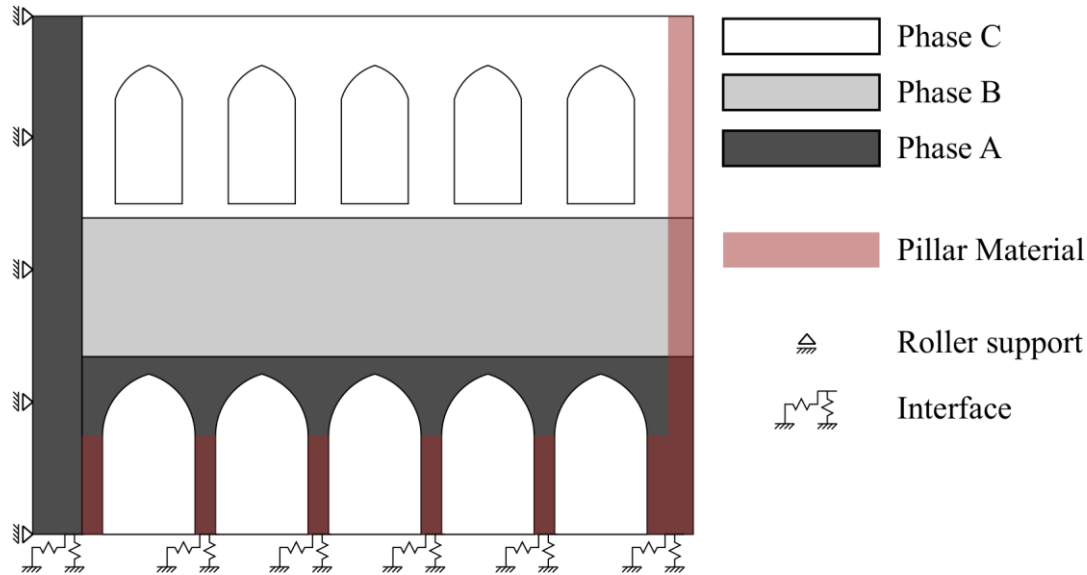
288 mode is, in any case, constrained by the high compressive stress applied on the interface by the self-weight and  
289 the lack of horizontal loads. Therefore, the shear stiffness is calculated for a Poisson's ratio of 0.20 without further  
290 investigation.

291 The soil beneath the church foundations has been investigated and found to be generally composed of, from  
292 the surface advancing in depth: a) sandy clay ( $\pm 2.0$  m thick), b) highly compressible peat ( $\pm 2.0$  m thick), c) sandy  
293 clay ( $\pm 2.6$  m thick), d) quaternary clay-containing sand ( $\pm 5.4$  m thick) and e) tertiary highly consolidated clay-  
294 containing sand (unknown thickness) [10]. The foundation bases of the main pillars are roughly in the middle of  
295 the peat layer. In the present case study, the settlements have been calculated prior to this investigation. Therefore,  
296 the interface normal stiffnesses can be directly calculated from eq. (5) and (4). Using eq. (3) the apparent Young's  
297 modulus of the homogenized foundation soil can be back-calculated. In the case of the nave pillars at the final  
298 settlement (as shown in Figure 4) this apparent Young's modulus is equal to  $0.597 \text{ N/mm}^2$  or  $2.400 \text{ N/mm}^2$   
299 when taking into account or disregarding the effect of embedment respectively. The former value is representative  
300 of peats, while the latter is rather low for the all the soils in the other layers. This indicates the major contribution  
301 of the peat layer to the total settlements and the potential primary cause of the excessive settlements of the church  
302 at the nave.

303 For all analysis cases, the structure is let to deform under its self-weight and the extra load applied at various  
304 levels of the nave wall from other elements present in the structure but not explicitly modeled, such as the timber  
305 roof or the stone masonry vaults.

306 All finite element calculations were carried out using the DIANA FEA package [22]. The geometric layout  
307 and the boundary conditions applied are illustrated in Figure 7. The different phases are colored in shades of grey  
308 and the areas where pillar masonry material is assigned are given a reddish overlay. It is assumed that the tower to  
309 the west of the nave (left side in the illustration) provides a rigid lateral support to the nave. This assumption is  
310 based on the greater bending stiffness of the tower, due to greater foundation depth, better preservation state, wall  
311 thickness and closed box plan, compared to that of the nave wall. It is further assumed that the nave wall is not  
312 constrained towards the crossing in the east. This assumption, in turn, is based on the connection to the transept

313 being effected by the vaults, which cannot provide significant constraint to a solid masonry wall. The average  
 314 element length is roughly 166 mm, resulting in a total of 40308 nodes and 13029 continuum and 37 interface  
 315 elements. The roof loads associated with each of the three phases is applied at the top of every model.

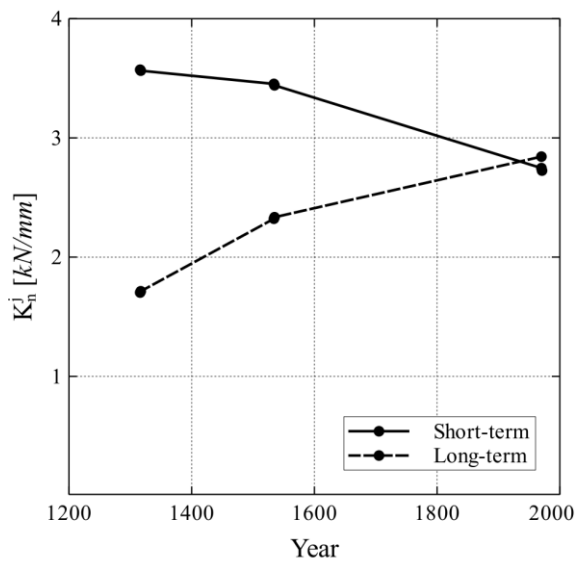


316  
 317 **Figure 7 Geometric layout of church nave wall. Structural phase designation, boundary conditions**  
 318 **and material assignment.**

### 319 3.3 Definition of Time Periods for Phased Analysis

320 Three different time periods are defined for the investigation of the time-dependent behavior of the building:  
 321 period A (1300-1316), period B (1317-1534) and period C (1535-1970), which coincide with the construction  
 322 phases illustrated in Figure 7. The beginning of period A corresponds to the completion of the first nave wall level.  
 323 The end of period A corresponds to the initiation of the increase of the height of the nave and period B starts upon  
 324 its completion. The end of period B corresponds to the construction of the crossing and the third level of the nave  
 325 wall. Period C brings us near to the present period, at the time of major temporary shoring of the building. The  
 326 analysis period covers the entire history of the monument up to before the point of internal shoring. The post-  
 327 intervention state of the monument will be a subject of further study in the future.

328 Through use of the equations (3) to (5), and for known values of the applied force and displacement, one may  
 329 calculate the normal interface stiffness, and also back-calculate the apparent Young's modulus of the soil at given  
 330 time instants. Both the forces and the settlements have been estimated for various points in the history of the  
 331 building, as shown in Figure 4. The calculated normal interface stiffness for different structural parts through time  
 332 is shown in Figure 8. This value is proportional to the apparent Young's modulus of the soil. One can differentiate  
 333 between a short-term Young's modulus, governed by immediate settlements due to a change in load, and a long-  
 334 term Young's modulus, governed by settlements due to, for example, consolidation. In the case of the pillars, the  
 335 short-term Young's modulus generally exhibits a decreasing trend, whereas the long-term modulus exhibits an  
 336 increase. The two curves appear to converge near the end of the measurement period, indicating that processes  
 337 causing settlement under sustained loads have been halted. The consolidation being further completed accounts  
 338 for the latter phenomenon, but the continuation of the settlements cannot be entirely excluded.



339

340 **Figure 8** Calculated pillar spring stiffness for estimated loads and settlements. Development of  
 341 instant and long-term response.

### 342 3.4 Analysis Approaches

343 Three different approaches will be adopted for the analysis of the nave: a single-phase analysis, a phased  
 344 analysis and a parametric study.

345 Firstly, in the single-phase analysis, the whole structure is taken in its entirety and the foundation interfaces  
346 are assigned their final values according to the estimated settlements of each structural part and the dimensions of  
347 the footings.

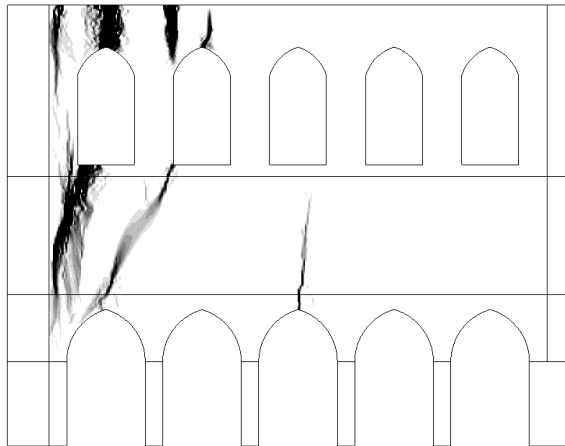
348 Secondly, in the phased analysis, three major phases are considered, with each one decomposed into two parts.  
349 The major phases correspond to the phase designation indicated in Figure 7 and the decomposition of each phase  
350 is based on the differences between the short-term (immediate) and the long-term response of the structure under  
351 the sustained loads of each phase. Both the single-phase analysis and phased analysis make use of approach (b)  
352 for determining the interface normal stiffness as explained in Section 3.2. In order to facilitate the conformity of  
353 the previously and newly active parts of the mesh during the transition between phases, the structure was unloaded  
354 in a stepwise manner before the activation of the new parts. This approach maintains the damage location and the  
355 local reduction of stiffness due to cracking.

356 The self-weight is applied in 50 steps for each phase of all the analyses. A regular Newton-Raphson iteration  
357 method is employed, with a 0.001 energy norm for convergence. Thirdly, a parametric analysis is performed to  
358 address uncertainties in the mechanical properties of the materials. In order to reduce the computational cost, these  
359 analyses are only carried out for the settlement profile obtained from the levelling surveys of the period 1994-  
360 2005. Such analysis is defined in Section 3.2 as approach (a) for determining the normal stiffness. In the following  
361 section, this choice of loading is motivated further, through a discussion of the obtained failure patterns. The  
362 material parameters included in the investigation are the Young's modulus, the tensile strength and the tensile  
363 fracture energy of the masonry composing the nave. The variation of the parameters ranged from 50% to 200% of  
364 the initial values indicated in Table 1. The model using the initial values will be henceforth referred to as the  
365 reference model.

## 366 4. Analysis Results

### 367 4.1 Single-Phase Analysis

368 As a first approach, the self-weight of the complete structure along with the final additional roof weights is  
369 applied in a single phase. The final value for the stiffness of the foundation interfaces is used (see Figure 8),  
370 corresponding to the long-term soil modulus of phase C. The obtained crack pattern is shown in Figure 9. The  
371 obtained damage pattern presents several differences from the actual structure. Due to the settlement towards the  
372 crossing, the response is dominated by the separation cracks between the nave and the tower. Crack 2 above arch  
373 2 (Figure 2) is entirely absent. A single crack is formed between pillars 2 and 3 (crack 3, but inclined in the other  
374 direction) and the crossing itself remains intact, as does its connection with the wall. Therefore, application of the  
375 deformation loads in a single analysis step reveals only part of the response of the building and is not indicative  
376 of its behavior throughout its history.



377  
378 **Figure 9 Crack patterns for application of load in single analysis phase.**

### 379 4.2 Phased Analysis

380 The phased analysis of the nave provides a much more complete and detailed illustration of the development  
381 of damage on the building throughout its history (Figure 10). The obtained settlement profiles derived from the  
382 phased analysis, as well as from the single-phase case, are presented in Figure 11.

383 During phase A, only slight damage is registered at arches 1 and 5, see Figure 10a. There is some widening of  
384 the cracks under long-term loading, but no formation of new major cracks, see Figure 10b. The response is mostly  
385 of a sagging type due to the tower and the crossing exhibiting only minor settlement. Due to the numerical cracks  
386 not having sufficiently developed in extent, no clear comparison between the numerically derived cracks and the  
387 existing damage in the building can be made.

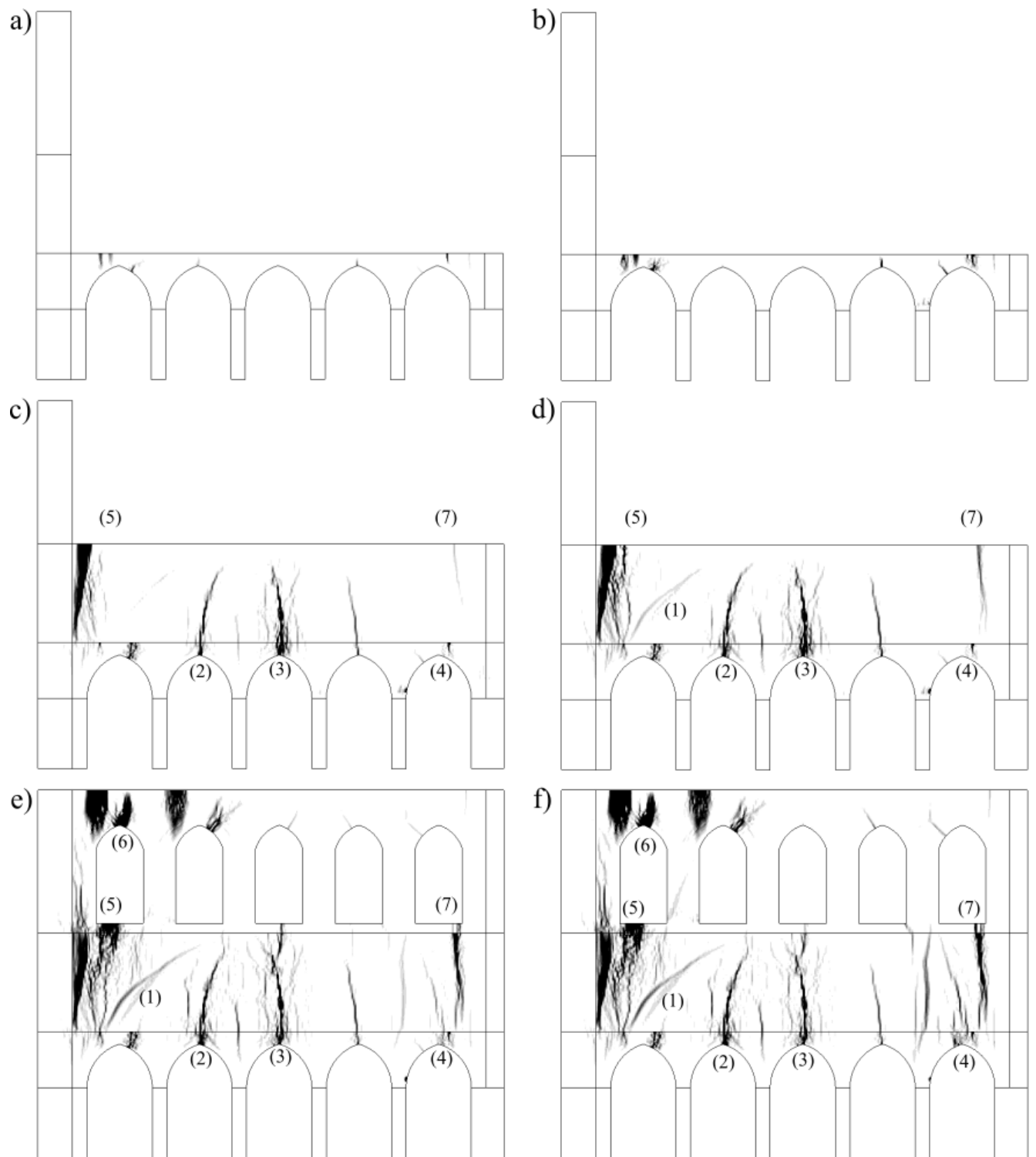
388 The situation changes significantly during phase B with the increase of the height of the nave wall above the  
389 colonnade. A major separation crack is formed in the short-term phase between the tower and the nave (cracks 1  
390 and 5). Additionally, cracks 2 and 3 are formed above pillar 3. At the end of the long-term loading, cracks 1 and  
391 4 have emerged above the main colonnade. While the settlement profile is mostly of a sagging type at the end of  
392 the short-term phase (Figure 11), a mixed profile with a significant tilting component is obtained at the end of the  
393 long-term phase. The increase of the weight at the crossing is substantial and unable to be borne by its foundations.

394 Phase C witnesses the formation of the new cracks above and beside the third level windows (cracks 6 and 7  
395 in Figure 2). The extent of the previously formed cracks is increased without, however, significant widening,  
396 indicating the activation of the cracks at the new parts of the structure. As shown in Figure 11, the obtained profile  
397 resembles the tilting-dominated response at the end of phase B, but of a larger magnitude.

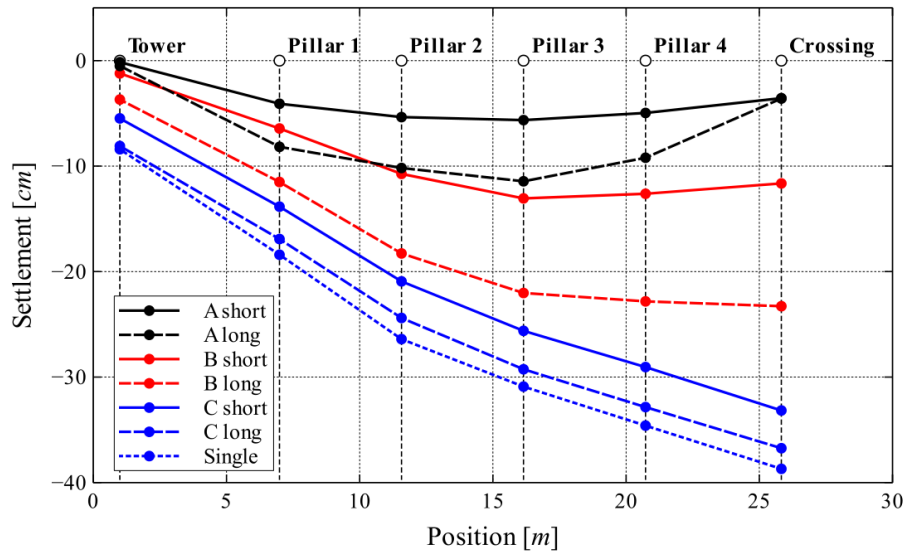
398 In Figure 11, the maximum settlement obtained at the end of the phase C closely resembles that of the single-  
399 phase analysis, albeit with a slightly smaller magnitude. Nevertheless, the cracking pattern is significantly different  
400 in the two approaches. The resulting cracking pattern from the phased analysis resembles in a higher degree the  
401 actual pattern (compare Figure 2 and Figure 10). The cracking pattern resulting from the phased analysis resembles  
402 much more closely the actual crack pattern compared to the single-phase analysis (compare Figure 9 and Figure  
403 10). The complexity of the model, as the outcome of soil-structure interaction, is underlined by the substantially  
404 different settlements obtained between the four pillars. This is despite the fact that they are of the same cross-  
405 section with identical foundations (meaning equal interface stiffness) and bearing roughly the same vertical loads.

406 Overall, as time progresses, the deflection ratio of the structure tends to decrease, despite the increase in the  
407 overall settlements (Figure 11). Cracks 2 and 3, caused by the sagging of the center are nearly fully developed by  
408 the end of phase B, after which newly arising damage is possibly associated with tilting of the nave towards the  
409 crossing. The maximum width of the cracks in the first level of the wall in fact decreases from phase B to phase C  
410 due to the stabilizing effect of the added stiffness of the second level and despite the increase in weight.



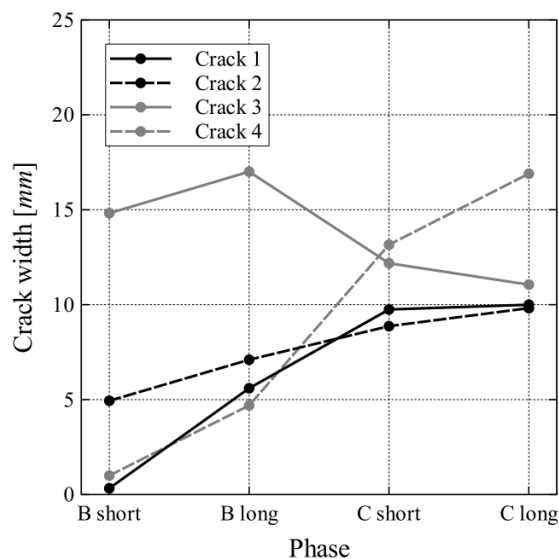


**Figure 10 Crack patterns for phased analysis of nave: a) phase A short-term, b) phase A long-term, c) phase B short-term, d) phase B long-term, e) phase C short-term, f) phase C long-term.**



**Figure 11 Settlement profiles obtained from finite element analysis: phased analysis and single-phase approaches.**

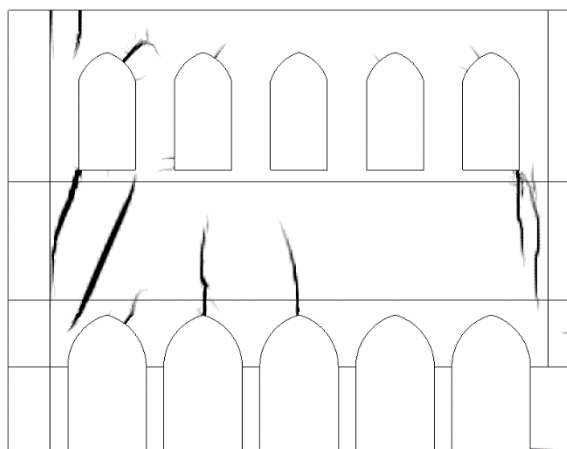
The numerically obtained cracks have been linked to cracks documented in the structure (Figure 2 and Figure 10). Rather than integrating the obtained crack strains over the continuum to calculate the crack width, the opening of the cracks is indirectly calculated through measurement of the horizontal relative displacements of nodes on either side of the smeared crack mouth. Displacements due to elastic stress are minimal compared to displacements due to crack opening. In cases where the cracks in the actual structure are composed of more than one distinct branch, this measured numerical crack width is divided by the number of branches in order to obtain the magnitude of a single crack branch. The development of the normalized crack width, defined as the sum of the crack width divided by the number of crack branches in the actual structure, is presented in Figure 12. All cracks tend to increase with the passage of time, except crack 3, which is reduced in width after the construction of the second and third level of the nave.



**Figure 12 Normalized crack width for various analysis phases.**

### 4.3 Parametric Investigation

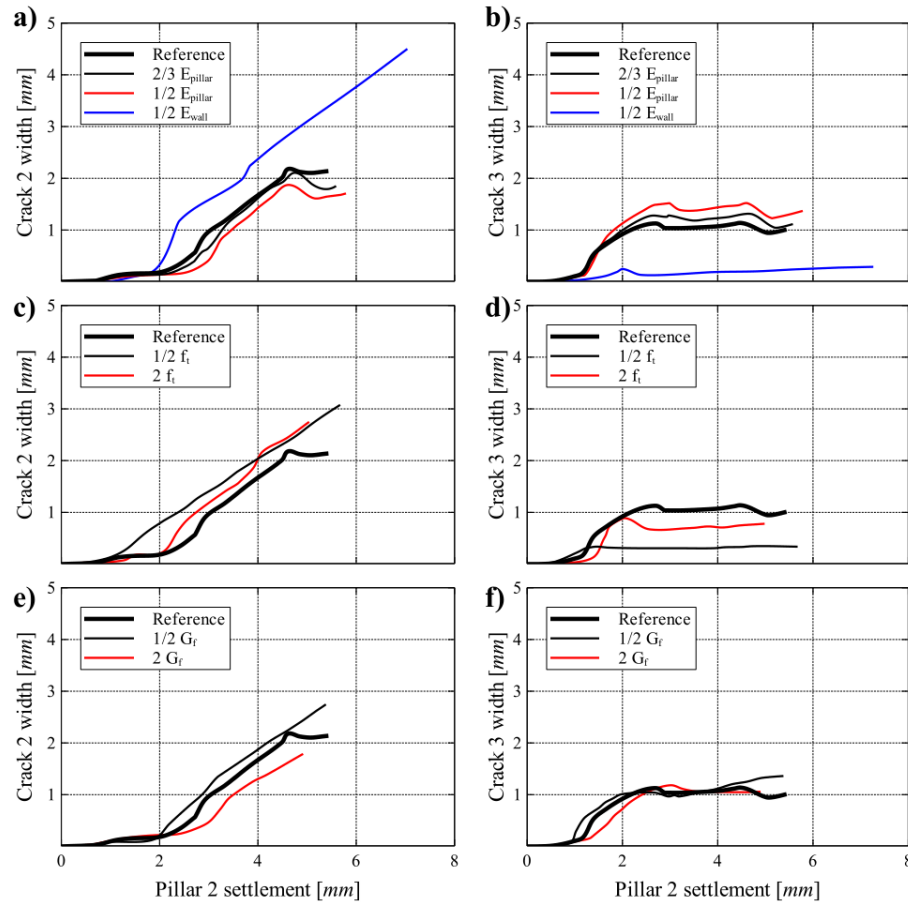
The damage pattern obtained from the reference model is shown in Figure 13. This is the outcome of the application of the settlement pattern measured in the period 1994-2005. Rather than the displacement being applied to the supports directly, the stiffness of the interfaces was calibrated in order to match this settlement profile. This approach allows the evaluation of the soil-structure-interaction by altering the settlements from a variation of the stiffness of the superstructure.



**Figure 13 Crack pattern obtained from the reference model of the parametric investigation.**

437 Despite the narrow extent of this measurement period compared to the entire history of the building, the  
438 pattern closely resembles the damage present in the structure, both in location and extent, albeit with a much  
439 smaller magnitude in terms of crack width. This constitutes an indication that the emergency measures taken from  
440 1965 onwards may have not completely halted the progress of settlement in the structure. Nevertheless, this  
441 resemblance motivates the use of this measured profile as a basis in the parametric investigation carried out in this  
442 section.

443 The results of the parametric investigation are illustrated in Figure 14. They are presented in terms of the width  
444 of cracks 2 and 3 vs. settlement of pillar 2, above which the cracks in question are situated. Initially, the Young's  
445 modulus of the pillar was investigated, due to the initially determined value being higher than expected given the  
446 compressive strength of the pillar masonry (see Table 1). The change in the Young's modulus of the pillar does  
447 not significantly affect the response of the nave, due to the limited extent of the nave area in which it is encountered.  
448 However, the reduction of the Young's modulus of the wall increases both the width of the cracks and the amount  
449 of settlement of pillar 2. Interestingly, the reduction of the Young's modulus causes crack 2 to increase in width  
450 and crack 3 to be severely reduced, owing to the redistribution of forces in the wall. Cracks 2 and 3 are differently  
451 affected by changes in the tensile strength of the wall masonry as well. The width of crack 2 slightly increases for  
452 any change in the parameter, while crack 3 decreases, practically disappearing for a decrease in the tensile strength.  
453 Finally, the response was not particularly sensitive to changes in the tensile fracture energy of the wall masonry.  
454 However, a slight increase in the total settlement of the pillar is registered for a decreased value of this parameter.



**Figure 14 Results of parametric investigation: a) & b) Crack width for variation of Young's modulus, c) & d) crack width for variation of tensile strength, e) & f) crack width for variation of tensile fracture energy.**

The results are also tabulated in Table 2, with the addition of the results of the horizontal movement and vertical settlement of pillar 2. The changes in the horizontal movement of the pillar, measured at the capital and indicating tilting rather than whole body slipping, are either associated with the crack widths above the pillar or with the capacity of the wall for post-cracking deformation (as influenced by an increase in the fracture energy).

Crack 2 did not present strong sensitivity to the material properties. Crack 3 presented some sensitivity to the tensile strength of masonry. The dependence on the Young's modulus of masonry is partially related to the increase of crack width due to reduced stiffness of the superstructure. The tensile fracture energy had only a marginal effect on either crack. The crack arrangement was not sensitive to the material properties. These observations suggest

that crack formation and development are more sensitive to the applied deformation profile. This outcome illustrates the importance of an accurate calculation of the properties of the soil and a good measurement of the historic settlement profile in order to achieve meaningful analysis results. In light of the envisaged intervention project, consolidation of the wall masonry, which would lead to some degree of increase of the tensile strength, fracture energy and Young's modulus of the material, can be beneficial in itself for maintaining the integrity of the church and limiting the effects of possible future differential settlement. This can act as a complement to the foundation strengthening and soil consolidation underway.

**Table 2 Results of parametric investigation. Percentile differences from reference model results in crack width and displacement (vertical and horizontal) of pillar 2.**

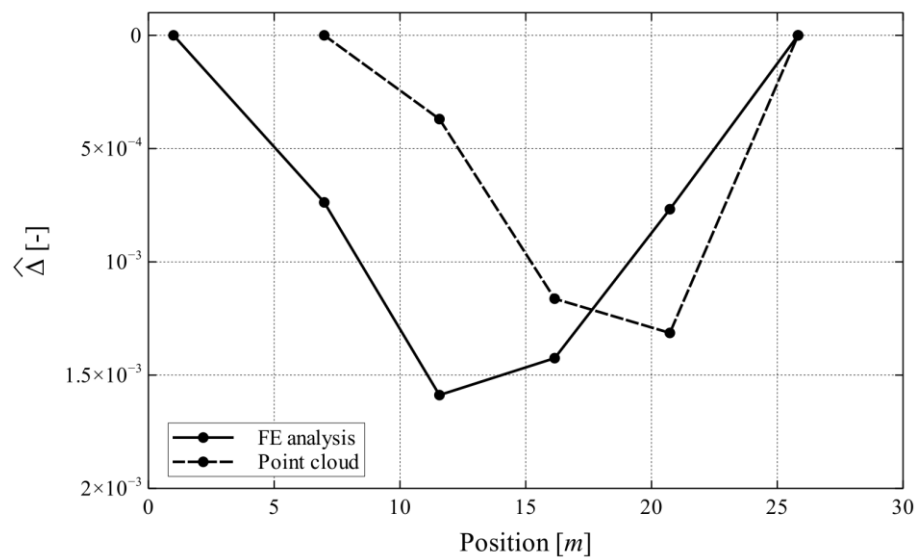
	Crack width 2	Crack width 3	$\Delta x_{pillar\ 2}$	$\Delta y_{pillar\ 2}$
Reference	100	100	100	100
$1/2 f_t$	146	32	190	105
$2f_t$	129	77	85	93
$1/2 G_f$	129	137	76	99
$2G_f$	83	105	271	91
$2/3 E_{pillar}$	85	111	97	103
$1/2 E_{pillar}$	80	136	102	107
$1/2 E_{wall}$	218	29	352	134

#### 4.4 Discussion on Model Results

A final appraisal of the obtained results from the modeling approaches is warranted. In particular, the results from the phased analysis need to be contrasted with the monitoring and survey data. The final deformation profile obtained for the phased analysis (Figure 11) relies on the estimation of the behavior of the soil through time. The profile estimated from the architectural features of the building (Figure 6) depicts the effects of soil-structure interaction more directly. The latter profile compares favorably with the deformation profile obtained from monitoring data over the period 1994-2005. The normalization of the estimated settlements by disregarding the tower movement prior to the construction of the nave wall makes a direct comparison difficult. The lack of measurements in the period 2005-2018 further complicates matters, which the new intervention project will help clarify. Finally, the analysis approach does not take into account the full three-dimensional geometry effects of the

486 structure. These are potentially more acute near the crossing pillar due to the presence of the transept. Nevertheless,  
487 the numerical analysis approach adopted here is able to capture the response of the nave with significant fidelity.

488       The assumption of a uniform interface stiffness below all the pillars in the nave is a necessary simplification  
489 due to the lack of more detailed data. Despite resulting in an accurate crack pattern, the final disposition is not in  
490 complete agreement with the settlement as estimated from the architectural feature analysis. However, a clearer  
491 relation between the numerical and measured deformation profile is obtained when comparing deflection ratios.  
492 The deflection ratio is defined as the ratio of the relative settlement to the length of the deflected part. In Figure  
493 15, the deflection ratios along feature Line 4 (base of the pillar abacus of the first level) are presented, based on  
494 the measurements shown in Figure 6. The deflection ratio along this line could be more clearly defined along a  
495 larger portion of the structure compared to other lines and is sufficiently close to the base of the pillars to provide  
496 an indication of the settlement. As Figure 15 illustrates, the deflection ratios between the two approaches are quite  
497 similar, with the sagging of the colonnade between the tower and the crossing pillar being clearly indicated.



499

500 **Figure 15 Comparison of deflection ratio from finite element analysis and architectural feature**  
501 **measurement on the point cloud (Line 4).**

502 **5. Damage Calculation Using an Analytical Damage Function**

503 **5.1 Calculation of Model Parameters**

504 The model proposed by Giardina et al [7], which introduced an analytical relation between settlement and  
505 damage index for masonry buildings, will be adopted for the present study. The model relates the deflection ratio  
506  $\hat{\Delta}$  due to sagging or hogging ground deformation to a damage level of the structure according to the classification  
507 proposed by Burland & Wroth [23]. The damage level, linked to the severity of damage and the means required  
508 for its repair, is quantitatively expressed in terms of crack width, thus directly comparable to both documented  
509 pathology and nonlinear finite element analysis results. The damage classes are outlined in Table 3. The damage  
510 model for two-dimensional structures is a function of several geometric and material parameters expressed in a  
511 polynomial equation as follows:



$$d'_{2D}(\hat{\Delta}, \bar{x}) = d_{2D,ref}(\hat{\Delta}) + \sum_{i=1}^6 a_i \bar{x}_i = b_1 + b_2 \hat{\Delta} + b_3 \hat{\Delta}^2 + b_4 \hat{\Delta}^3 + \sum_{i=1}^6 a_i \bar{x}_i \quad (7)$$

where  $d_{2D,ref}$  are the selected reference values,  $a_i$  and  $b_i$  are fitted polynomial coefficients and  $\bar{x}$  contains the normalized values of the model parameters  $x_i$ . The  $x_i$  model parameters, along with their reference values  $x_{i,ref}$  are given in Table 4. All values for the polynomial coefficients and the normalization process for the model parameters are detailed in [7]. From the value of the damage level calculated from the model, one can calculate the corresponding crack width through linear interpolation based on the values found in Table 3.

**Table 3 Damage classification for masonry structures subjected to differential settlements [23].**

Damage Level	Damage Class	Crack Width [mm]
1	Negligible	0.0 – 0.1
2	Very Slight	0.1 – 1.0
3	Slight	1.0 – 5.0
4	Moderate	5.0 – 15.0
5	Severe	15.0 – 25.0
6	Very Severe	>25.0

**Table 4 Damage model reference values  $x_{i,ref}$  [7] and input for current analysis  $x_i$ .**

Openings	$G_f$	$E$	$f_t$	$k_n$	Interface shear behavior/Trough shape
[%]	[N/m]	[N/mm <sup>2</sup> ]	[N/mm <sup>2</sup> ]	[N/mm <sup>3</sup> ]	[-]
$x_{1,ref} = 30$	$x_{2,ref} = 10$	$x_{3,ref} = 3000$	$x_{4,ref} = 0.10$	$x_{5,ref} = 0.7 \times 10^9$	$x_{6,ref} = 1$
$x_1 = 31.92 \div 33.88$	$x_2 = 12$	$x_3 = 3000$	$x_4 = 0.10$	$x_5 = [\text{See Table 5}]$	$x_6 = 1$

Parameters  $x_1$  to  $x_4$  are derived according to the material properties used in the finite element analysis and the geometry of the nave. Parameter  $x_6$  is assigned its reference value according to [7]. Special attention is drawn to the parameter  $x_5$  related to the normal stiffness of the soil-structure interface. The reference value for  $x_5$  has been calculated according to typical Dutch pile foundation systems distributed along the façade of brick masonry structures [24]. In the present research, this parameter is calculated from the modulus of subgrade reaction under vertical loading of the foundation system of the nave, an approach that generalizes the applicability of the damage function to other foundation and soil types. This approach additionally allows for taking into account foundation strengthening, micro-piling and foundation soil improvement directly in the damage function.

527 For strip foundations, the modulus of subgrade reaction  $k_n$  is directly equivalent to the parameter  $x_5$  and can  
 528 be applied to continuous shallow foundations of masonry walls. Equations for its calculation are available in the  
 529 literature (e.g. [25,26]). Some further manipulation is required in the case of colonnades founded on individual  
 530 footings, as is the case with the nave pillars of the present case study. The  $x_5$  parameter for a series of  $m$  single  
 531 footings  $j$  is then calculated as follows:

$$k_n = \frac{\sum_{j=1}^m k_n^j A_f^j}{\sum_{j=1}^m A_f^j} = x_5 \quad (8)$$

532 where  $k_n^j$  is the modulus of subgrade reaction of footing  $j$ . This can be calculated from equations (3) and (4)  
 533 or from equations (4) and (5) if the settlement has been pre-estimated. This averaging approach to the subgrade  
 534 reaction modulus is similar to the one followed for the allocation of the stiffness provided by the distributed piles  
 535 according to Rots [24], but can be generalized as shown for continuous footings of walls or isolated footings of  
 536 colonnades. This parameter becomes significant in light of the results shown in Figure 11. Despite the decrease in  
 537 the deflection ratio as the phases progress, the damage, in terms of crack width, increases. This is captured by the  
 538 damage model through the change in the  $x_5$  parameter due to soil-structure interaction (decrease of the apparent  
 539 Young's modulus of the foundation soil within a single phase). The disposition of the data points, capable of being  
 540 approximated by a third order polynomial fit, suggests that the damage model can be successfully adapted to this  
 541 case.

542 In addition to adjusting the reference value for the vertical interface stiffness ( $x_5$ ), the  $a_5$  coefficient associated  
 543 with the interface stiffness is also adapted. These parameters are modified in order to fit the available numerical  
 544 results of the phased analysis (phase B and C) and the reference model. The material properties of masonry are  
 545 taken as equal to those of the wall masonry (Table 1), which comprises most of the structure and on which the  
 546 majority of the damage is accumulated. The percentage of openings varies between 33.88% in phase B and 31.92%  
 547 in phase C and the reference model.

## 5.2 Results

Using equation (8) the values for the parameter  $x_5$  are found for the colonnade. These are shown in Table 5, where it becomes apparent that the range of the parameter values varies within the range initially investigated in [7], with the exception of the reference model case. Nevertheless, it is expected that the calibrated values for the numerical model parameters be significantly different from those initially proposed. This is due to the fact that the initial model was calibrated against a finite element benchmark where the interface stiffness was not extensively investigated. A complete recalibration of the model is bound to alter the numerical parameters to some degree.

The slight increase of  $x_5$  during the transition from long-term phase B to short-term phase C is contrary to the progress of soil consolidation, which decreases the apparent short-term Young's modulus of the soil. However, depending on the calculation method elected (such as for the estimated settlement method employed here or for continuous footings), the  $x_5$  parameter may depend directly or indirectly on the stiffness of the superstructure as well. This stiffness is increased by the addition of the second level of the nave wall. This fact clearly illustrates the significance of soil-structure interaction in the study of differential settlement damage problems.

The analytical model, when used with its initial reference values and normalization process, greatly exaggerates the damage corresponding to the reference model and underestimates the damage in the phased analysis (phases B and C). The calibration of the new parameters is performed by a simple minimization process, during which the linear regression between FE and analytical crack widths is required to be a unitary slope curve. The normalization of the  $x_5$  parameter in [7] is carried out according to:

$$\overline{x_5} = \frac{\log_{10} x_5 - \log_{10} x_{5,ref}}{2} \quad (9)$$

A new normalization of the parameters is proposed here in order to match the trend of interface stiffness to damage level, according to which the  $\overline{x_5}$  parameter is equal to:

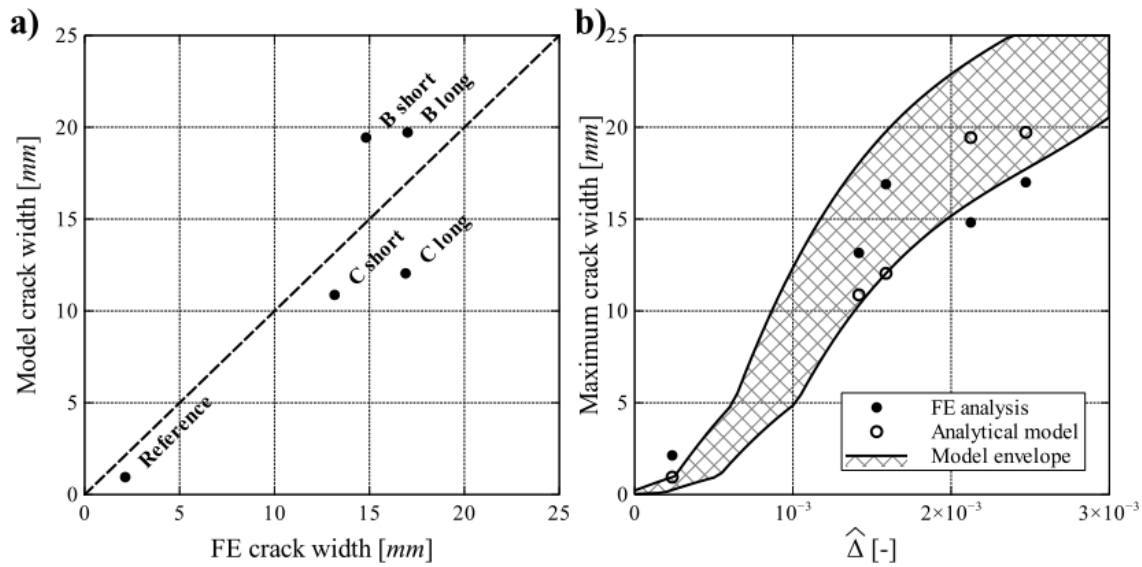
$$\overline{x_5} = \left( \frac{\log_{10} x_{5,ref}}{\log_{10} x_5} \right)^{1.5} \quad (10)$$

568 The results of the modified analytical model are compared with the finite element analysis results in terms of  
 569 the main cracks of the first level of the nave wall, following their calculation as described in the phased analysis  
 570 section. The minimization process for the available data set produces a value for the  $a_5$  parameter equal to -1.1722.  
 571 The comparison of the results is plotted in Figure 16a, in which satisfactory agreement is found throughout the  
 572 range of available data. Despite some discrepancy in the results in terms of crack width, the damage level is well  
 573 approximated by the calibrated model.

574 **Table 5  $x_5$  parameter results for phase B model, phase C model and reference model.**

	Reference	B short	B long	C short	C long
$x_5$ [N/mm <sup>3</sup> ]	1.43E+10	2.79E+07	9.63E+06	9.70E+06	6.86E+06

575 The envelope indicating the change in the polynomial model curve, almost entirely due to alterations in the  
 576 spring stiffness and to a very minor extent due to changes in the opening percentage, is presented in Figure 16b.  
 577 The upper envelope curve corresponds to the maximum apparent stiffness associated with the reference model and  
 578 the lower curve corresponds to the minimum apparent stiffness of the long-term part of phase C. This fact  
 579 illustrates the influence of the foundation stiffness, and, by extension, the properties of the soil, on the behavior of  
 580 complex structures.



**Figure 16 a) Comparison of analytical model with FE analysis results. Dashed diagonal indicates line of equality between FE analysis and analytical modeling. b) Crack width according to damage prediction model and comparison with FE analysis results.**

The initial results of this approach towards the extension of the predictive analytical model are promising. The generalization of this extension requires robust verification in order to recalculate the numerical coefficients of the model. Coupled experimental tests and parametric numerical analyses need to be developed, along the same lines of the prior development of the analytical model but attempting to include variations in the vertical stiffness of the foundation. This is likely to lead to modifications to all numerical parameters, unlike the simplified calibration approach adopted in this paper, where only the parameters linked with the vertical stiffness were modified.

## 6. Conclusions

In this study the behavior of a church nave wall subjected to ground deformation over an extended period of time is investigated. The problem is approached through single-phase and multi-phase finite element analysis. The paper demonstrates the importance of detailed modeling of the soil behavior over time, the soil-structure interaction and the accurate measurement of settlements for the analysis of complex structures subjected to soil movement. Through this investigation it is shown that the application of a single-phase analysis does not reliably provide the cracking pattern observed in the actual structure. The need to take into account construction phases

598 and changes in the soil stiffness is clearly shown, even when studying individual element ensembles, such as the  
599 nave wall here investigated. The importance of detailed geometric and damage survey is also demonstrated.

600 The sensitivity analysis illustrates the predominance of the deformation profile, as influenced mainly by the  
601 soil-structure interaction, in the disposition of the cracking pattern on the structure. Nevertheless, the width of the  
602 cracks is strongly influenced by the material properties of the nave wall, as are the obtained settlements, although  
603 to a lesser degree.

604 The phased analysis, taking into account changes in the behavior of the soil and alterations in the geometry of  
605 the structure, provides a complete picture of the history of the nave's pathology. Different major cracks appear  
606 and develop at different phases of the building, due to redistribution of the forces and changes in the stiffness of  
607 the foundations.

608 An analytical model for the prediction of the damage level in masonry structures subjected to differential  
609 settlements is adapted and expanded. Moving beyond the initial formulation of the model, a method for the direct  
610 calculation of the normal stiffness of the structure is proposed. Following calibration of the numerical coefficients  
611 linked to this stiffness, the results of the model are consistent with the numerical analysis results and the crack  
612 state of the real structure. Further expansion of the model along the lines pursued here can greatly enhance the  
613 potential for accurate analytical modeling of complex masonry structures.

## 614 **Acknowledgements**

615 The authors acknowledge the funding received by BRAIN.be, Belspo in support of the GEPATAR research  
616 project ("GEotechnical and Patrimonial Archives Toolbox for ARchitectural conservation in Belgium"  
617 BR/132/A6/Gepatar).

## 618 **References**

619 [1] A.R. Sánchez, R. Meli, M.M. Chávez, Structural Monitoring of the Mexico City Cathedral (1990-2014),  
620 Int. J. Archit. Herit. 10 (2016) 254–268. doi:10.1080/15583058.2015.1113332.

- 621 [2] L. Giresini, Energy-based method for identifying vulnerable macro-elements in historic masonry churches,  
622 Bull. Earthq. Eng. 14 (2016) 919–942. doi:10.1007/s10518-015-9854-7.
- 623 [3] F. Portioli, L. Cascini, Large displacement analysis of dry-jointed masonry structures subjected to  
624 settlements using rigid block modelling, Eng. Struct. 148 (2017) 485–496.  
625 doi:10.1016/j.engstruct.2017.06.073.
- 626 [4] S. Galassi, G. Misseri, L. Rovero, G. Tempesta, Failure modes prediction of masonry voussoir arches on  
627 moving supports, Eng. Struct. 173 (2018) 706–717. doi:10.1016/j.engstruct.2018.07.015.
- 628 [5] P. Roca, M. Cervera, L. Pelà, R. Clemente, M. Chiumenti, Continuum FE models for the analysis of  
629 Mallorca Cathedral, Eng. Struct. 46 (2013) 653–670. doi:10.1016/j.engstruct.2012.08.005.
- 630 [6] S. Saloustros, L. Pelà, P. Roca, J. Portal, Numerical analysis of structural damage in the church of the  
631 Poblet Monastery, Eng. Fail. Anal. 48 (2015) 41–61. doi:10.1016/j.engfailanal.2014.10.015.
- 632 [7] G. Giardina, M. Hendriks, J.G. Rots, Damage Functions for the Vulnerability Assessment of Masonry  
633 Buildings Subjected to Tunneling, J. Struct. Eng. 141 (2015) 1–13. doi:10.1061/(ASCE)ST.1943-  
634 541X.0001162.
- 635 [8] D. Peduto, G. Nicodemo, J. Maccabiani, S. Ferlisi, Multi-scale analysis of settlement-induced building  
636 damage using damage surveys and DInSAR data: A case study in The Netherlands, Eng. Geol. 218 (2017)  
637 117–133. doi:10.1016/j.enggeo.2016.12.018.
- 638 [9] D. Peduto, S. Ferlisi, G. Nicodemo, D. Reale, G. Pisciotta, G. Gullà, Empirical fragility and vulnerability  
639 curves for buildings exposed to slow-moving landslides at medium and large scales, Landslides. 14 (2017)  
640 1993–2007. doi:10.1007/s10346-017-0826-7.
- 641 [10] L. Schueremans, K. Van Balen, K. Brosens, D. Van Gemert, P. Smars, The Church of Saint-James at  
642 Leuven: Structural Assessment and Consolidation Measures, Int. J. Archit. Herit. 1 (2007) 82–107.

- doi:10.1080/15583050601126137.
- [11] S. Sanchez-Beitia, L. Schueremans, K. Van Balen, On-site stress measurement on the piers of the Saint Jacobs church in Leuven, Belgium, *Int. J. Archit. Herit.* 3 (2009) 110–125. doi:10.1080/15583050802278794.
- [12] E. Verstrynge, L. Schueremans, P. Smars, Controlled Intervention: Monitoring the Dismantlement and Reconstruction of the Flying Buttresses of Two Gothic Churches, *Int. J. Archit. Herit.* 6 (2012) 689–708.
- [13] G. Heirman, K. Brosens, Uitvoeringsdossier Globale Stabiliteitswerken Sint-Jacobskerk. Internal report D/0800/10, Triconsult nv, 2017.
- [14] M. Bassier, G. Hardy, L.-E. Bejarano-Urrego, A. Drougkas, E. Verstrynge, K. Van Balen, M. Vergauwen, Semi-automated creation of accurate FEM meshes of heritage masonry walls from point cloud data, in: 11th Int. Conf. Struct. Anal. Hist. Constr. Cusco, Peru, 2019: pp. 305–314.
- [15] K. Van Balen, K. Nuyts, P. Smars, D. Van de Vijver, Optimalisatie van standzekerheidsmodellen van gewelfde gotische structuren gebruik makend van informatie uit vervormingsmetingen en scheuranalyse. Internal report PV30318, KU Leuven, 1995.
- [16] A.W. Koppejan, A formula combining the Terzaghi load compression relationship and the Buisman secular time effect, in: *Proc. 2nd Int. Conf. Soil Mech. Found. Eng.*, 1948: pp. 32–38.
- [17] G. Giardina, A. V van de Graaf, M. Hendriks, J.G. Rots, A. Marini, Numerical analysis of a masonry façade subject to tunnelling-induced settlements, *Eng. Struct.* 54 (2013) 234–247. doi:10.1016/j.engstruct.2013.03.055.
- [18] A. Drougkas, P. Roca, C. Molins, Numerical prediction of the behavior, strength and elasticity of masonry in compression, *Eng. Struct.* 90 (2015) 15–28. doi:10.1016/j.engstruct.2015.02.011.
- [19] R. De Borst, Smeared cracking, plasticity, creep, and thermal loading—A unified approach, *Comput.*



665           Methods Appl. Mech. Eng. 62 (1987) 89–110.

666   [20]   H.W. Reinhardt, Fracture Mechanics of an Elastic Softening Material Like Concrete., *Heron*. 29 (1984) 1–  
667           42. doi:10.1023/A:1012235530463.

668   [21]   G. Gazetas, Foundation Vibrations, in: H.-Y. Fang (Ed.), *Found. Eng. Handb.*, 1991: pp. 553–593.  
669           doi:10.1007/978-1-4615-3928-5\_15.

670   [22]   TNO, DIANA Finite Element Analysis, User’s Manual, (2017).

671   [23]   J.B. Burland, C.P. Wroth, Settlement of buildings and associated damage, in: *Settl. Struct. Proc. Conf. Br.*  
672           *Geotech. Soc.*, 1974: pp. 611–764.

673   [24]   J.G. Rots, Settlement Damage Predictions For Masonry, in: *Maint. Restrengthening Mater. Struct. Brick*  
674           *Brickwork Proc. Int. Work. Urban Herit. Build. Maint. IV*, 2000: pp. 47–62.

675   [25]   M.A. Biot, Bending of an infinite beam on an elastic foundation, *J. Appl. Math. Mech.* 2 (1937) 165–184.  
676           doi:10.1016/0021-8928(58)90136-9.

677   [26]   A.B. Vesić, Beams on Elastic Subgrade and the Winkler’s Hypothesis, in: *Proc. 5th Int. Conf. Soil Mech.*  
678           *Found. Eng. Paris, Fr.*, 1961: pp. 845–850.

679